SEISMIC PERFORMANCE EVALUATION OF MULTISTORIED RC BUILDING USING PUSHOVER ANALYSIS WITH DISPLACEMENT COEFFICIENT AND CAPACITY SPECTRUM METHODS

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Abstract

In the realm of civil engineering, the seismic response of reinforced concrete is one of the crucial aspects that may ensure structural safety under seismic loading. This study evaluates the seismic response of the four-storied RC moment-resisting frame under the guidance of FEMA-273, FEMA-356, and ATC-40. The structure was modeled in ETABS with plastic hinges at the beam and column ends, considering material and geometric nonlinearities (P-δ, P-Δ effects). Capacity curve and hinge progression were asserted to analyse the performance level, i.e., Life Safety (LS), Collapse Prevention (CP), and Immediate Occupancy (IO). To validate the results, manual calculations were done in parallelly using the Capacity Spectrum Method (CSM) and the Displacement Coefficient Method (DCM) as per code ASCE 41-17. The evaluated results emphasise the intensity of consistency when compared to the software outputs, and additionally indicate the framework that exhibits adequate ductility and energy dissipation, satisfying IO and LS under design-level earthquakes and CP under maximum considered events. The framework confirms that pushover analysis, when complemented by simplified manual methods, provides a practical and robust approach for assessing seismic performance and planning retrofits of RC buildings. The outcomes of this study not only reinforce the reliability of nonlinear static procedures but also underscore their relevance in performance-based seismic design frameworks.

Keywords: Pushover Analysis, Displacement Coefficient Method, Nonlinear Static Analysis, Capacity Spectrum Method

1 Introduction

Seismic resilient design in a structure is one of the most crucial aspects, especially in earthquake-prone areas. In the past 90s, engineers used methods to assume the elasticity of the structure, so that it can absorb earthquake energy without getting damaged. However, the greater earthquakes, like the 1994 Northridge earthquake in the USA and the Kobe earthquake in Japan, proved them wrong by leaving back devastating impact on the cities of the USA and Japan, proving that traditional methods are not always safe enough [1]. Many buildings that met code requirements were still badly damaged or even collapsed during these events.

Mentioning the above problem, a new method was employed called performance-based seismic design (PBSD). The method especially focuses on the actual response of the building subjected to the ground motion. This approach is taken to set a limit on how much damage is acceptable, such as making sure the building can still be used after a small earthquake or that it will not collapse during a very strong one. These goals are usually defined as performance levels mentioned: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). To check the accurate response of the building, the time history method is employed. But the method is known to be lot of time-consuming as well as large amount of data, which is not always practical. The perfect alternative to the analysis is pushover analysis, used widely by the engineers [2]. In this method, the building is pushed sideways, and then the deformation and damage are measured.

The article aims to demonstrate the non-linear static pushover analysis in ETABS. It employs both the Displacement Coefficient Method (DCM) as well as Capacity Spectrum Method (CSM). According to the guidelines of FEMA and ASCE code. Throughout the process, automated modelling techniques are compared with manual calculations to assess seismic performance under Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) levels. Plastic hinge development, performance point, and capacity curves were analysed to validate structural safety and guide retrofit decisions.

2 Methodology

2.1 Fundamental principles

The pushover analysis was done to evaluate the seismic performance of a multi-story reinforced building under gradually increasing lateral loads, simulating earthquake effects.

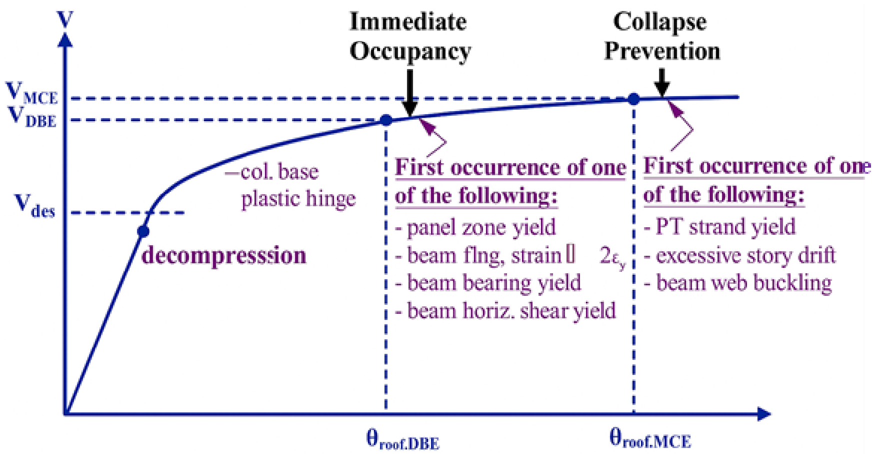
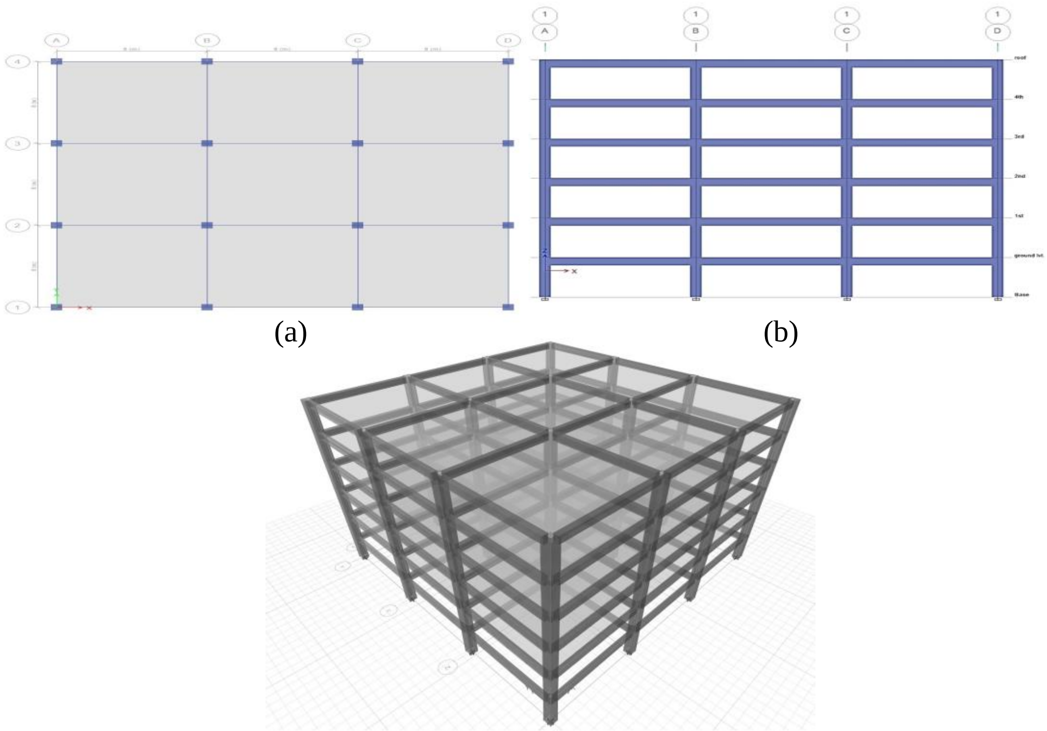


Fig. 1. Idealised pushover curve showing base shear versus roof drift with DBE/MCE thresholds and performance levels

This nonlinear static approach can be tracked by the plastic hinges and generates capacity curves, especially base shear vs roof displacement, to assess the structure's ability to withstand seismic forces while meeting performance-based seismic design (PBSD) objectives such as Immediate Occupancy, Life Safety, and Collapse Prevention mentioned in Fig. 1.
The Four-Storied structure is modelled using ETABS with a regular type of structure as per codal provisions, IS 1893- 2016, IS 456- 2000, and IS 1875 [5], [8], [9]. The material used for construction is reinforced concrete with M-30 grade concrete and Fe500 grade reinforcing steel. The Stress-Strain relationship used is as per I.S.456:2000 [8]. The basic material properties used are as follows:

Modulus of Elasticity of steel (E_S) = 20,0000 MPa
Modulus of Elasticity of concrete (E_C) = 25,000 MPa
Characteristic strength of concrete, (f_{ck}) = 30 MPa
Yield stress for steel, (f_Y) = 500 MPa
Ultimate strain in bending, (ε_{cu}) = 0.
The structure analysed is a five-story structure, as shown in Fig. 5c, consisting of four bays along the X-direction and four bays along the Y-direction, an ordinary moment-resisting frame of reinforced concrete with properties as specified above. The concrete floors are modelled as rigid. Seismic mass source (DL+0.5LL) for analysis P-delta load case (DL+0.5LL)

2.2 Details of the models are given as



(a) (b)

(c)

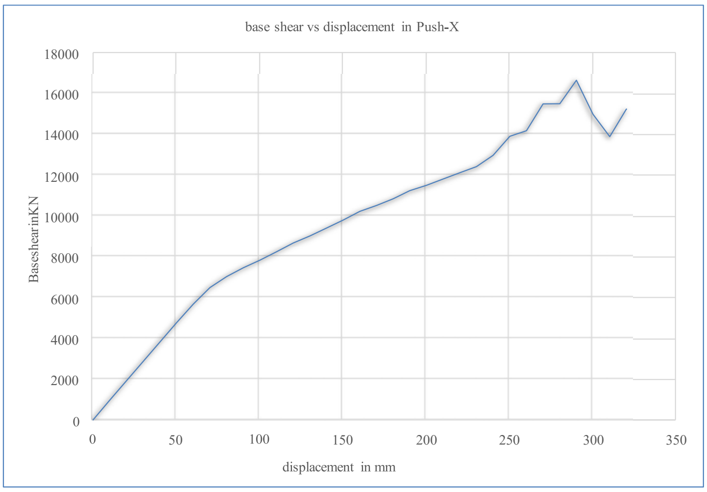
Fig. 2. ETABS Model Plan view, elevation view & 3D view

The dimension, Hinge and material properties reflect the model in Fig. 2, Number of stories = 5 , Storey height = 3.0 meters , Bay width of each node along X-direction = 8.0 meters, Bay width of each node along the Y-direction = 8.0 meters, Column size = 600x600, beam size = 300x600, Concrete grade = M30, Rebar material = HYSD500, slab thickness = 165mm, Reinforcement steel is provided in column = 1.4%, Reinforcement provided in beam for Pushover Analysis = 0.8% , Displacement and control parameters were designed according to the code ASCE 41-17 [7], Hinge type - Interacting P-M2-M3 (Coupled hinge), Hysteresis type – Isotropic, Immediate occupancy – 0.01rad, Life safety – 0.025 rad, Collapse prevention – 0.05 rad.

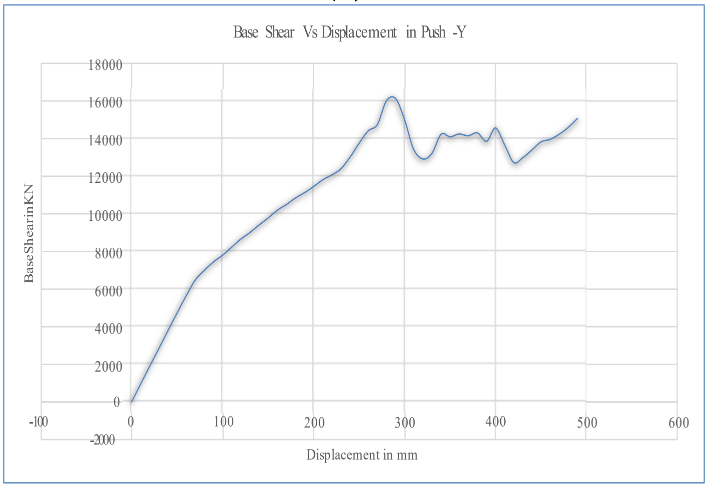
3 Results and discussions

3.1 Analysis in ETABS

The pushover analysis curve represents the relationship between the base shear and roof displacement, illustrating the structural transitions from elastic to inelastic behaviour. The base shear vs. displacement graph under increasing lateral load is shown in Fig. 3.



(a)



(b)

Fig. 3. hot spot stress area and maximum principle stress (X&Y direction)

3.2 Manual calculations

Variables of displacement modification method (DMM) as per ASCE 41-17
V_Y - yield base shear = 7024 kN (from pushover curve)
V_u - ultimate base shear = 15000 kN (from pushover curve)
D_Y - yield displacement (Roof displacement) = 80.179 mm (pushover)
D_u - ultimate displacement = 400 mm (pushover)
V_d - design base shear = 4515 kN (code / design-level base shear)
T' - (effective first-mode period used for DMM) = 1.083 s (from modal analysis)
C_0 = 1.4 (code coefficient used in ASCE 41 DMM)
C_1 = 1.00316 (Inelastic displacement ratio specified in ASCE 41-17)
C_2 = 1.00012 (Period dependency factor specified in ASCE 41-17)
C_m (effective mass) = 1 (mode participation factor used in their calculation)
w = seismic weight = 34436 kN (model weight)

Response reduction factor = 7.76115
g = 9.8 gravitational acceleration (used implicitly when converting g units)
Derived quantities

X_W (yield strength coefficient) = $V = \frac{V_Y}{W}$ (1)

M (demand-to-capacity ratio at yield)

$M = \frac{C_m \times S_a}{V_y \times X_W}$

Overstrength factor (Ω0)

$= \frac{V_y}{V_d} = \frac{7024}{4215} = 1.5557$ (2)

Ductility ratio (μ) (This μ is the ductility ratio, taken from ASCE 41 tables/curves.)

$\frac{D_u}{D_y} = \frac{400}{80.179} = 4.9888$ (3)

S_a (peak acceleration for Design Basis Earthquake DBE) $= Z \times 2f\left(\frac{S_a}{g}\right)$ (4)

S_a (peak acceleration for maximum considered earthquake MCE) $= Z \times f\left(\frac{S_a}{g}\right)$ (5)

3.3 Comparison of manual calculation and analytical curve

Comparison of manual calculation with displacement Y direction as per ASCE 41-17. From Eqn. 4, we get,

S_a (peak acceleration for DBE) $= Z \times 2f\left(\frac{S_a}{g}\right)$ = 0.272g (6)

Target displacement by the DMM method

$D_t = C_0 \times C_1 \times C_2 \times C_3 \times S_a \times \frac{T^2}{4x} \times \pi^2 g$

D_t = 111.4615 mm

S_a = (peak acceleration for MCE) $= Z \times f\left(\frac{S_a}{g}\right)$ (7)

D_t = 226.372 mm.

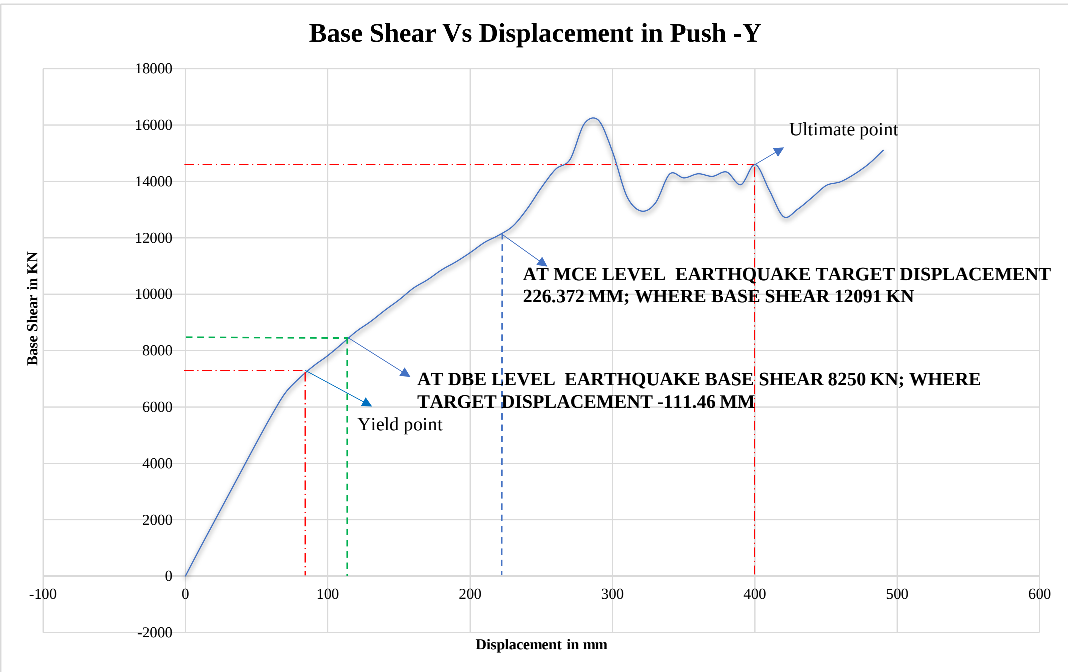
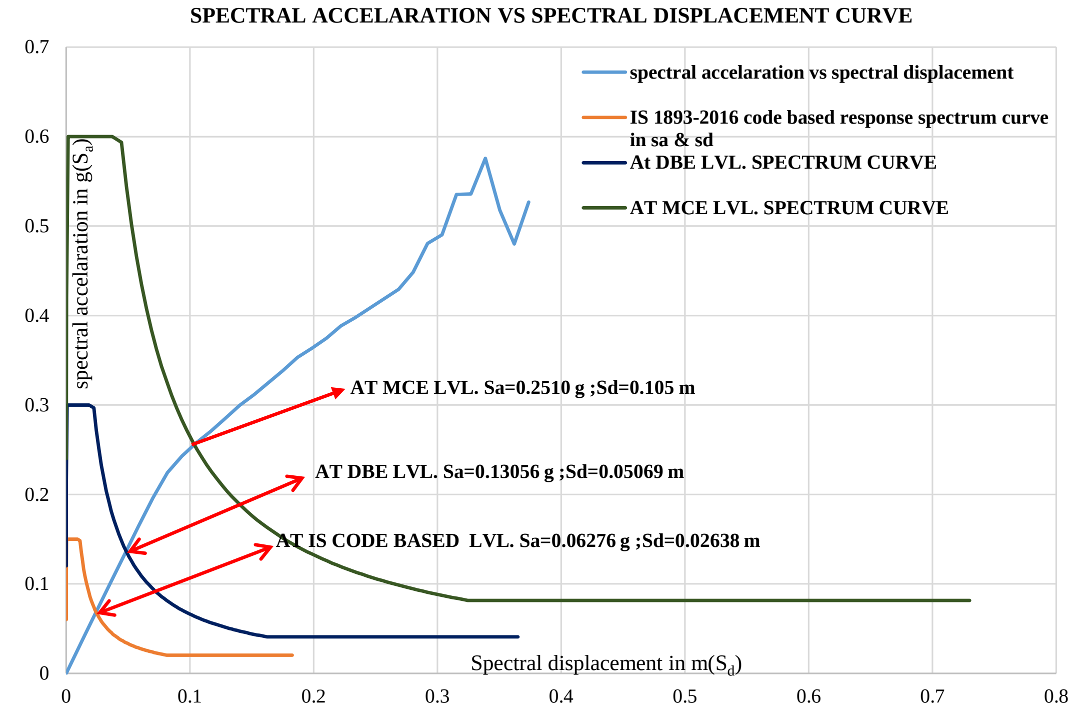
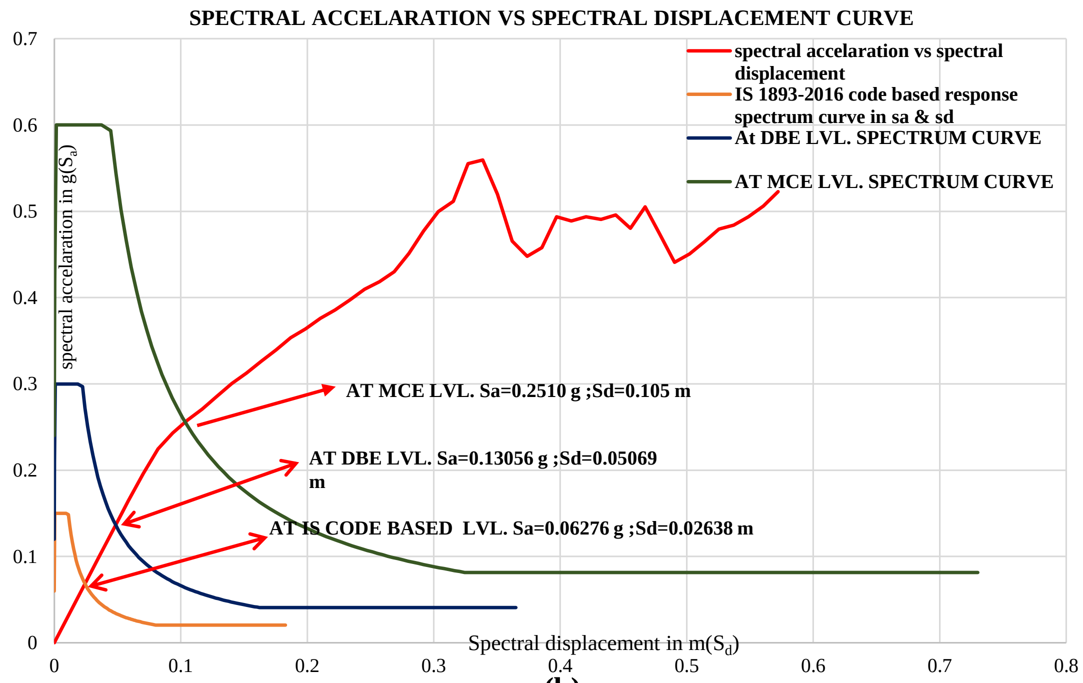


Fig. 4. Push Curve along Y direction showing Target displacement at DBE & MCE Level

3.4 Seismic Performance Evaluation by Capacity Spectrum Method



(a)



(b)

Fig. 5. Spectral acceleration–spectral displacement relationships for PUSH-Y (a) and PUSH-X (b) directions, showing ATC-40 and IS 1893 code-based spectra with MCE and DBE performance points from nonlinear pushover analysis

4. Observation

4.1 Performance point identified and calculation process

IS Code-Based Point: Sa = 0.06276 g; Sd = 0.02638 m (Indicates performance under standard design conditions); DBE Level Point (Design Basis Earthquake): Sa = 0.13056 g; Sd = 0.05069 m (Represents moderate intensity earthquake response); MCE Level Point (Maximum Considered Earthquake): Sa = 0.2510 g; Sd = 0.105 m (Represents extreme seismic event; performance here is crucial).

Target Spectrum as per IS 1893-2016: $S_d = \frac{S_a \times T}{39.478}$

Calculation of SDOF displacement: $S_d - \frac{Displacement\ in\ mm}{1000 - T}$

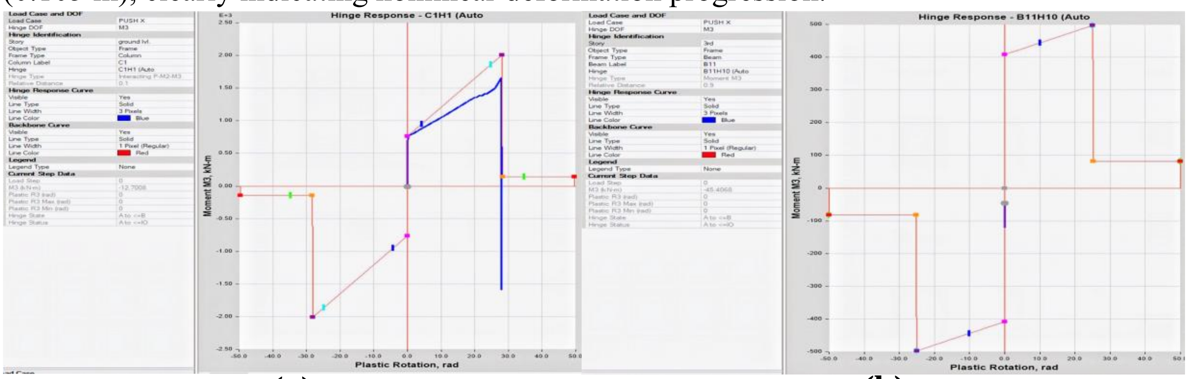
Calculation of SDOF acceleration: $S_a - \frac{V_{base}}{TM^{*} \times 9.81}$

4.2 Curve interpretation

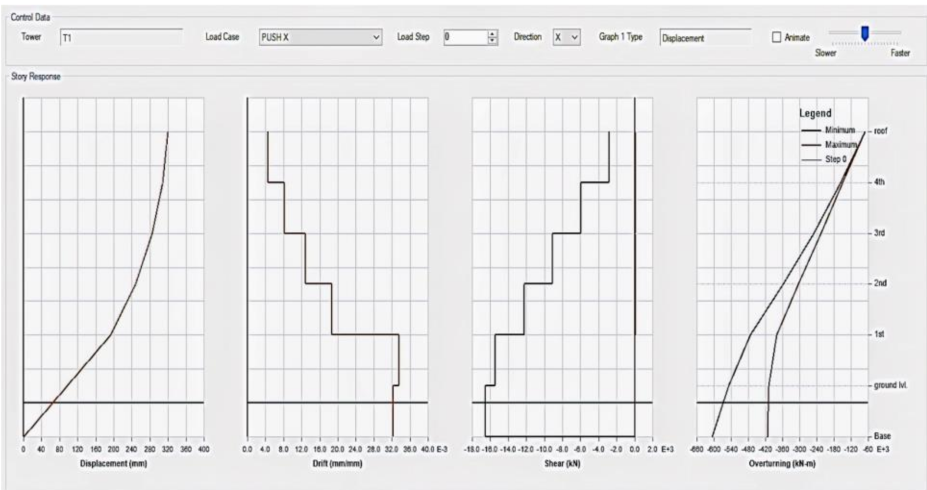
The red (or blue) curve in the respective figures represents the pushover (capacity) curve obtained from nonlinear static analysis; The green and navy-blue curves represent demand spectra at MCE and DBE levels, respectively; The orange curve follows IS 1893:2016 provisions, showing idealised code-based

4.3 Displacement trends

The displacement increases from IS level (0.02638 m) - DBE (0.05069 m) - MCE (0.105 m), clearly indicating nonlinear deformation progression.



(a) (b)



(c)

Fig. 6. (a) Push -X loading Column Hinge result, (b) for Push-X loading Beam Hinge Result, (c) For Push-X loading Displacement, Drift, Storey shear, Overturning moment Curve from Etabs

4.4 Observation of Hinge result for Beam, column and frame displacement, drift, etc.

Nonlinear pushover analysis showed a stable and ductile response in both beams and columns under lateral loads.

Column hinge C1H1 (ground-level, column C1) developed plastic rotation ±0.028 radians, confirming controlled inelastic action.

Moment capacity of C1H1 reached ±2004 kN·m, indicating strong flexural strength and high energy dissipation.

Response curve followed backbone curve closely, confirming stable post-yield behaviours with no major strength degradation.

C1H1 met FEMA-356 Collapse Prevention (CP) criteria, maintaining structural integrity under extreme demand.

Beam hinge B11H10 (3rd floor, 90% span) showed plastic rotation of ±0.025 radians, validating good ductility.

The moment capacity of B11H10 was ±497 kN·m, demonstrating effective reinforcement and design.

Hinge curve matched backbone shape, suggesting proper detailing and material performance.

B11H10 also met the CP level with no brittle failure or sudden strength drop, ensuring safe energy dissipation.

4. Conclusion

The analysis of a multi-storied reinforced concrete (RC) building was carried on under two hazard levels, Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE), using both the Displacement Coefficient Method (DCM) and the Capacity Spectrum Method (CSM). The guidelines that were followed throughout the analysis were FEMA 356, ASCE 41-17, and ATC-40.

The curves of base shear vs. roof displacement revealed a transition from the elastic to inelastic range, showing sufficient energy dissipation capacity and ductility. The plastic hinges formed at the beam ends, progressively developed in columns at lower stories, validating the strong column–weak beam design philosophy.

Under MCE conditions, most hinges remained Life Safety (LS) performance levels, while under DBE most hinges remained within Immediate Occupancy (IO), hinges approached but did not exceed the Collapse Prevention (CP) state. The target displacements computed from both DCM and CSM methods showed close relations, confirming the reliability and consistency of the adopted modelling approach. The structure exhibited stable hysteretic behaviour, moderate stiffness degradation, and no sign of global instability or soft-story formation. The results demonstrate that the RC frame has adequate strength and ductility to resist seismic demands without collapse, thereby ensuring the safety of occupants during strong ground motions.

Overall, the study confirms that pushover analysis is a robust and practical approach for performance-based seismic design and assessment. It effectively captures nonlinear behaviour, identifies potential failure mechanisms, and provides a clear framework for evaluating and improving the seismic resilience of RC structures.

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